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FLOW STRUCTURE IN A COMPOUND CHANNEL WITH SMOOTH AND ROUGH FLOODPLAINS

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ABSTRACT

Compound channels are a common configuration of rivers. During extreme events of floods, the momentum transfer due to the difference of the velocities between the main channel and the floodplains flows generates a complex 3D flow. Accurate estimation of channel capacity remains a difficult issue.

Although several studies have being carried out in the past, a new experimental study aims revisiting some of the previous experiments in a facility with separated upstream water supply.

The experimental facility consists in a 10 m long, 0.4 m wide and 0.1 m high main channel in the centre of two symmetrical 0.7 wide floodplains. The transition between the subsections is made by banks with 45º slope. The slope of the flume bottom is 0.0011 m/m. The original bottom is hydraulically smooth boundary made of polished concrete. Half of the experiments were done with the floodplains covered by artificial grass (rough boundary). Besides other measurement instruments, a Vectrino ADV allowed the measurement of streamwise and spanwise velocity components, turbulence intensities and Reynolds shear stresses.

In order to avoid the mass transfer in the beginning of the channel, the upstream water supply is separated between main channel and floodplains taking into account recommendations presented in recent literature.

Four different flow conditions were tested, corresponding to uniform flows for relative depths (ratio of the water depths in the floodplain and in the main channel) approximately equal to 0.15 and 0.3, for smooth and rough floodplains.

The influence of the relative depth and the floodplain roughness is evaluated and some of the flow characteristics are presented. It includes the lateral distributions of streamwise velocity and Reynolds stresses.

Finally, the accuracy of the total cross-section discharge obtained by several 1D methods is assessed. These methods were the Divided Channel Method, the Coherence Method, the Integrating Divided Channel Method, the Weighting Divided Channel Method and the Exchange Discharge Method.

1 INTRODUCTION

During floods the main channel of rivers may not be enough to convey the total discharge and a compound channel configuration can occur. In these cases, the flow submerges the surrounding fields, called the floodplains. The difference of the water depth and the bottom roughness between the main channel and the floodplains generates a difference in the streamwise velocity between these subsections. The faster flow in the main channel interacts with the slower flow in the floodplains generating a mixing layer near the interface. The cross section of compound channel rivers can be divided in two uniform zones and a mixing zone (Prooijen *et al.* 2005). This mixing region reduces the discharge capacity when compared with independent cross sections.

Water depths in single channels are accurately estimated since the method proposed by Antoine de Chézy (Myers 1978). This is not the case for compound channels, because of the velocity gradient between the flows in the main channel and in the floodplains, where the water depth is lower and, in many cases, the roughness is higher. This gradient generates a mixing layer in the interface which creates a 3D flow structure (Shiono and Knight 1991).

In many cases the floodplains are covered by vegetation, increasing the bottom roughness and the overall resistance. This difference leads to an increase of the velocity gradient between main channel and floodplain flows. Strong lateral shear layers between these regions are observed (Tang and Knight 2009).

The traditional method to study the flood inundation is based in an old approach that simply divides the total cross section with vertical divisions in the interface of the main channel and the floodplains. Besides that, new 1D approaches can take into account the interaction between the flows in each subsection. The 2D and 3D methods include some of the characteristics of compound channels. In engineering, due to the amount of data required and the processing time, 1D methods are often preferred. Still, the momentum transfer should be taken into account in 1D modeling (Bousmar and Zech 1999).

Since Sellin (1964) presented the first evidences of the flow characteristics in compound channels that there have been attempts to modeling it. Knight and Shiono (1996) referred the difficulty of the developed formulas to be applied universally as, in many cases, they had been set based on a reduced amount of data.

This paper intends to improve the knowledge of the flow in compound channels. Firstly, the flow structure in a compound channel is characterized for smooth and rough floodplains. For each case two different water depths are presented. Secondly, the accuracy of several 1D methods, available in the literature, is assessed for the four flow conditions.

2 ONE DIMENSIONAL METHODS

Modeling the flow in a compound channel as a simple channel by applying a formula of resistance to flow does not take into account the sub-section velocity differences. Chow (1959) suggested the division of the channel in subsections where velocity and roughness could be considered as uniform. This method, called the Divided Channel Method, is still widely used in commercial models as HEC-RAS (Brunner 2008), ISIS (Knight 2001), SOBEK and Mike 11 (Huthoff *et al.* 2008).

As pointed out in Knight (2001) this treatment of a compound channel assumes that there is no interaction between the subdivided areas despite the existence of mean velocity discontinuities at the assumed internal boundaries. Therefore the simple division of the channel in subsections is not appropriate for modelling the discharge in compound channels (Knight and Shiono 1996).

Different methods had been proposed with the attempt to model the interaction processes that occur in this type of flows, including the momentum transfer.

According to Knight (2001), these methods can be divided into 5 groups: i) methods that change the sub-area wetted perimeters; ii) methods that make discharge adjustments (with the experimental data, for example); iii) methods that include apparent shear stresses on the sub-area division lines; iv) methods where the lines are located at zero shear stress; v) methods that combine different divisions of the channel.

In this work, six methods were used to modelling the flow in the compound channel. Firstly, we used the two traditional methods called Divided Channel Method (DCM). From the groups presented before, we used the Coherence Method (CM) and the Debord Method (DM) from the group ii), the Exchange Discharge Method (EDM) and the Interacting Divided Channel Method (IDCM) from the group iii) and the Weighted Divided Channel Method (WDCM) from the group v).

A simple explanation of the calculation of the stage – discharge curves by each method is presented herein.

Divided Channel Method (DCM)

This method proposes the division of the channel in three sub-sections, namely the main channel and the lateral floodplains. The typical division is through vertical lines, where the total flow is given by the sum of sub-section discharges (*cf.* Eq. 1).

$$Q = \sum_i Q_i = \sum_i K_i R_i^{2/3} A_i S_0^{1/2} \tag{1}$$

in which Q stands for the discharge; K for the subsection roughness coefficient; R for the hydraulic radius; A for the cross section area and S_0 for the slope of the channel. Index i indicates each subsection.

Coherence Method (CM)

The Coherence Method was developed by Ackers (1993) and it improves the results of the DCM. This method uses two empirical coefficients for the adjustment of the sub-section discharges. The coherence (COH) is the relationship between the discharge obtained by the Eq. (1) but assuming only one section (Single Discharge Method - SCM, average roughness coefficient and velocity for the whole cross section) and the DCM (*cf.* Eq. 2).

$$COH = \frac{Q^{SCM}}{Q^{DCM}} \tag{2}$$

The closer to 1 is this coefficient, the more appropriate is to treat the channel as a single one. When this coefficient is significantly less than 1 it is necessary to apply a different coefficient, called DISADF (*cf.* Fig. 1) in order to correct the discharge in each subsection. An analysis of the experimental results has split the flow in 4 regions according to the relative depth (floodplain/main channel water depth ratio) of each one.

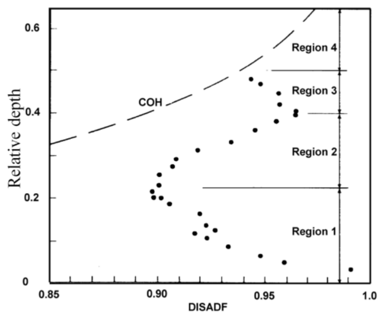

Figure 1. DISADF coefficient.

Ackers (1993) presented the formulas for computing the DISADF in each flow region. The discharge is then obtained by the following equations.

$$Q = Q^{DCM} - DISDEF \qquad \text{For flow region 1} \qquad (3)$$

$$Q = Q^{DCM} \times DISADF \qquad \text{For flow regions 2 to 4} \qquad (4)$$

in which DISDEF is a factor called discharge deficit which calculation procedure can be found, for example, in Wark *et al.* 1994.

Debord Method (DM)

The Debord Method proposes a correction of the DCM results based on experimental results conducted with 16 different configurations (Nicollet and Uan 1979). In those tests, the compound channel flow was compared with the flow in the independent sections (vertical separations were placed in the interface). The authors concluded that the most important parameter was the roughness ratio between subsections. The discharges can be computed with the Eqs. (5) and (6).

$$Q_{mc} = \varphi K_{mc} R_{mc}^{2/3} A_{mc} S_0^{1/2} \qquad (5)$$

$$Q_{fp} = \sqrt{1 + \frac{A_{mc}}{A_{fp}}\left(1-\varphi^2\right)}\, K_{fp} R_{fp}^{2/3} A_{fp} S_0^{1/2} \qquad (6)$$

in which subscripts *mc* and *fp* stand for main channel and floodplain, respectively and φ stands for the experimental coefficient given by:

$$\varphi = \varphi_0 = 0{,}9\left(K_{mc}/K_{fp}\right)^{1/6} \qquad \text{for } R_{fp}/R_{mc} > 0{,}3 \qquad (7)$$

$$\varphi = \frac{1}{2}\left[\left(1-\varphi_0\right)\cos\left(\frac{\pi\; R_{fp}/R_{mc}}{0{,}3}\right) + \left(1+\varphi_0\right)\right] \qquad \text{for } 0 < R_{fp}/R_{mc} \le 0{,}3 \qquad (8)$$

Exchange Discharge Method (EDM)

This method takes into account the concept of the apparent shear stress. The basis of this method is the transverse integration of the equation of momentum conservation.

After some simplifications and mathematical operations this equation could be written for the main channel and for the floodplains as showed in Eq. 9 and 10.

$$\rho \cdot g \cdot A_{mc} \cdot S_o + \left(h_{\text{int},rig} \cdot \tau_{\text{int},rig} + h_{\text{int},lef} \cdot \tau_{\text{int},lef}\right) - \tau_o \cdot P_{mc} = 0 \quad \text{Main channel} \quad (9)$$

$$\rho \cdot g \cdot A_{fp} \cdot S_o - h_{\text{int}} \cdot \tau_{\text{int}} - \tau_o \cdot P_{fp} = 0 \quad \text{Floodplains} \quad (10)$$

in which ρ stands for the density of water; g acceleration due to gravity; h_{int} – interface height; τ_{int} – apparent shear stress in the main channel and floodplain interface; τ_0 – bottom shear stress; P – wet perimeter; "*rig*" – right; "*lef*" – left.

To modeling the bottom shear stress it is only necessary to know the value of the apparent shear stress to calculate the rating curve of a compound channel.

EDM models the "momentum transfer due to turbulence" through a model similar to the mixing layer model (Smart 1992), resulting Eq. (11) for apparent shear stress (Bousmar and Zech 1999).

$$\tau_{\text{int}} = \frac{1}{2}\psi\rho\left(U_{mc} - U_{fp}\right)^2 \quad (11)$$

in which ψ stands for an experimental parameter and U stands for average velocity in a single subsection. EDM also models the momentum transfer associated with the geometry like converging main channels, which is out of the scope of this work.

Interacting Divided Channel Method (IDCM)

Huthoff *et al*. (2008) developed this method based in the apparent shear stress concept (Eq. 9 and 10). The authors used the formulation of Van Prooijen *et al.* (2005) in order to model the momentum transfer in the interface, obtaining the Eq. (12).

$$\tau_{\text{int}} = \frac{1}{2}\gamma\rho\left(U_{mc}^2 - U_{fp}^2\right) \quad (12)$$

in which γ corresponds to a coefficient obtained from experimental results collected in literature.

Weighted Divided Channel Method, WDCM

The Weighted Divided Channel Method was developed by Lambert and Myers (1998) and it is based on the observation of the velocity distributions in the main channel and floodplains. This method corrects the DCM results by weighting the velocities obtained with vertical and horizontal divisions between the subsections (Eqs. 13 and 14).

$$U_{mc} = \xi U_{mc}^{DCM-V} + (1-\xi)U_{mc}^{DCM-H} \quad (13)$$

$$U_{fp} = \xi U_{fp}^{DCM-V} + (1-\xi)U_{fp}^{DCM-H} \quad (14)$$

in which "*DCM-V*" stands for the results of DCM with vertical divisions; "*DCM-H*" with horizontal divisions and ξ for the weighting coefficient for the WDCM (from the experiments of the authors for equal roughness of the subsections the value of this coefficient is 0.5, see Lambert and Myers (1998) for further details.

3 EXPERIMENTAL DATA ACQUISITION

3.1 Experimental setup and equipment

The experiments were conducted in a compound channel located in the National Laboratory for Civil Engineering, in Lisbon. The experimental facility consists in a 10 m long, 0.4 m wide and 0.1 m high main channel in the centre of two symmetrical 0.7 wide floodplains. The slope of the flume bottom is 0.0011 m/m. The transition between the subsections is made by banks with 45º slope. Fig. 2 shows a schematic top view and cross-section of the flume.

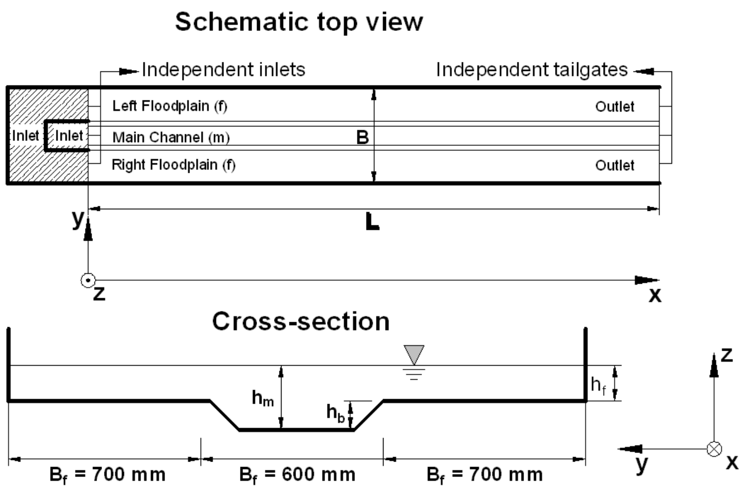

Figure 2. Schematic top view and cross-section.

The channel bottom is made of polished concrete. The rough experiments were done with the floodplains covered by commercial artificial grass. The roughness of the two channel bottoms (polished concrete and artificial grass) were estimated by the results of experiments with flow in a single channel. The Manning roughness coefficient for the polished concrete and for the artificial grass are 0.0095 s.m$^{-1/3}$ and 0.017 s.m$^{-1/3}$, respectively. The equivalent sand roughness k_s of the artificial grass is 0.00617 m and the average value of $u^* k_s/\upsilon$ (where u^* is the friction velocity in the centre of the channel and υ is the cinematic viscosity) is equal to 178 being a hydraulically rough boundary ($u^* k_s/\upsilon$ >70). The equivalent values for the polished concrete are k_s = 0.15 mm and $u^* k_s/\upsilon$ = 4.6 being a hydraulically smooth boundary.

Following the recommendations of Bousmar *et al.* (2005), separate inlets for the main channel and for the floodplains were installed in order to avoid the mass transfer between subsections. The discharges for the main channel and floodplains were monitored by two flowmeters and controlled by two different valves. Honeycomb diffusers and polystyrene plates were located at the beginning of the flume to stabilize the flow.

The flow regime is subcritical and the water depths were controlled by three independent tailgates located at the downstream end of the channel.

Water levels were measured with three point gauges, two of them fixed at the upstream and downstream sections of the flume and the other one place on a movable trolley allowing the measurement of the water depth in the entire channel. Streamwise velocity measurements were made using a Pitot tube with a 3.2 mm external diameter. The difference between static and dynamic pressures was measured with a differential pressure transducer.

The 2D and 3D velocity components were measured by an Acoustic Doppler Velocimeter, namely a side looking Vectrino. The acquisition time is 180s for each measurement, with a sampling frequency of 100 Hz. The sampling volume is a 7 mm long and 6 mm diameter cylinder. Taking into account the results obtained by the Pitot tube measurements, namely the symmetry of the discharge, the measurements with the Vectrino were done only in one half of the cross-section.

3.2 Experimental procedure

For the presented compound channel, the exact distribution of the discharge to the main channel and the floodplains was not known *a priori*. The procedure used to obtain an uniform distribution starts with the distribution given by the Weighted Divided Channel Method (Lambert 1998). With this first discharge distribution, the water levels were controlled with the tailgates in order to achieve a uniform water depth along the channel. When the water depth was constant, the discharge distribution at the measurement section (7.5 m from upstream) is compared with the upstream distribution. If the upstream and downstream discharge distributions match within less than 0.1 l/s, the uniform regime is considered to have been achieved. Otherwise the measured discharge distribution is imposed upstream and the procedure is repeated in an iterative process.

The streamwise velocities were measured with the Pitot tube in 45 verticals with 5 or 6 points each for the floodplains and main channel, respectively (*cf*. Fig. 3).

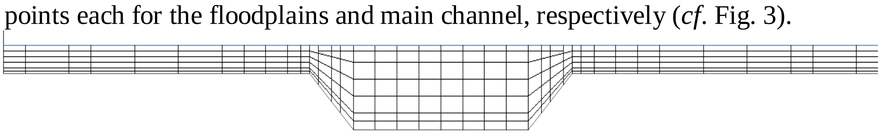

Figure 3. Mesh for the velocity measurements.

The measurements of the 2D/3D velocity components were performed for the uniform flow in the same verticals as presented in Figure 3 with 3 points in the floodplains and 7 points in the main channel. For the present work, four flow conditions have been adopted, namely the two different relative depths for each floodplain roughness. Table 1 presents these experimental conditions (FP and MC stand for floodplain and main channel, respectively).

Table 1. Experimental conditions.

Flow reference	MC depth, h_m (m)	Relative depth, h_r (-)	FP bottom	Discharge (l/s)		
				MC	FP	Total
hr015s	0.1172	0.15	Polished concrete	38.2	6.6	44.8
hr03s	0.1422	0.30	Polished concrete	54.2	26.4	80.6
hr015r	0.1192	0.15	Artificial grass	35.1	3.7	38.8
hr03r	0.1450	0.30	Artificial grass	42.3	16.6	58.9

4 Flow and Turbulence structure

4.1 Streamwise velocity

The streamwise velocity was measured with the Pitot tube in the positions presented in the mesh of Fig. 3. In Fig. 4 the isolines of the normalized streamwise velocity (u/u_m, where u_m represented the cross section average velocity) are presented for the two relative depths (h_r=0.15 and h_r=0.3) and for the case of smooth and rough floodplains.

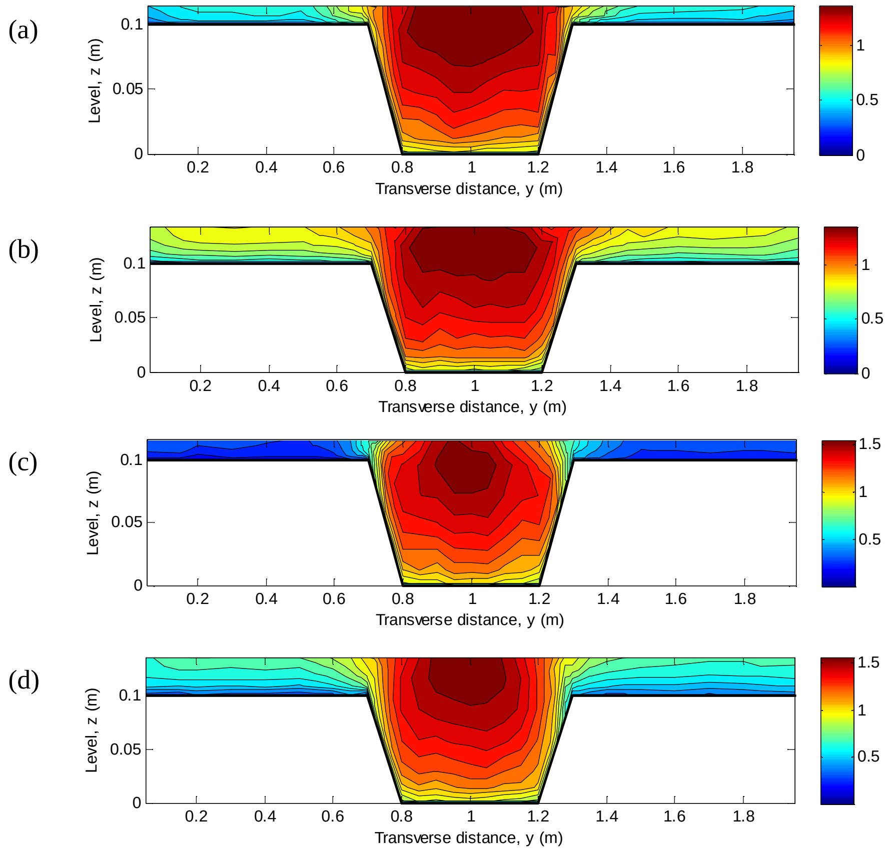

Figure 4. Normalized streamwise velocity (u/u_m). (a) hr015s; (b) hr03s; (c) hr015r and (d) hr03r

All the plots in Fig. 4 show the influence of the floodplain in the main channel flow namely the decrease of the velocity in the vicinity of the floodplains. The opposite occurs in the floodplain flow where an increase of the velocity is observed due to the presence of the main channel flow.

Two different comparisons can be made concerning the results presented in Fig. 4: i) the influence of the relative depth and ii) the influence of the floodplains roughness.

The increase of the water depth from h_r=0,15 to h_r=0,3 (comparing Fig. 4a with Fig. 4b and Fig. 4c with Fig. 4d) leads to a reduction of the interaction between these flows due to the decrease of the velocity gradient between the subsections. Regarding the isovels, there is a slightly difference between the two relative depths. Nevertheless, the overall distribution is similar.

The influence of the floodplain roughness on the isovels distribution is more evident. A lateral shift of the maximum velocity zone is observed for both relative depths. In average the velocity gradient is higher for the rough floodplain boundary.

4.2 Reynolds Stresses

The 2D and 3D vectrino measurements allow the calculation of turbulence intensities and Reynolds stresses.

In this paper, Reynolds stresses mean $\tau_{xy} = -\rho u' v'$, where ρ is the water density and u' and v' are the fluctuation velocities for the streamwise and spanwise directions, respectively.

In the Fig. 5 the isolines of the Reynolds stresses τ_{xy} are presented. They reveal the interaction between the flow in the floodplain and in the main channel. For all cases, the higher Reynolds stresses values are observed near the interface.

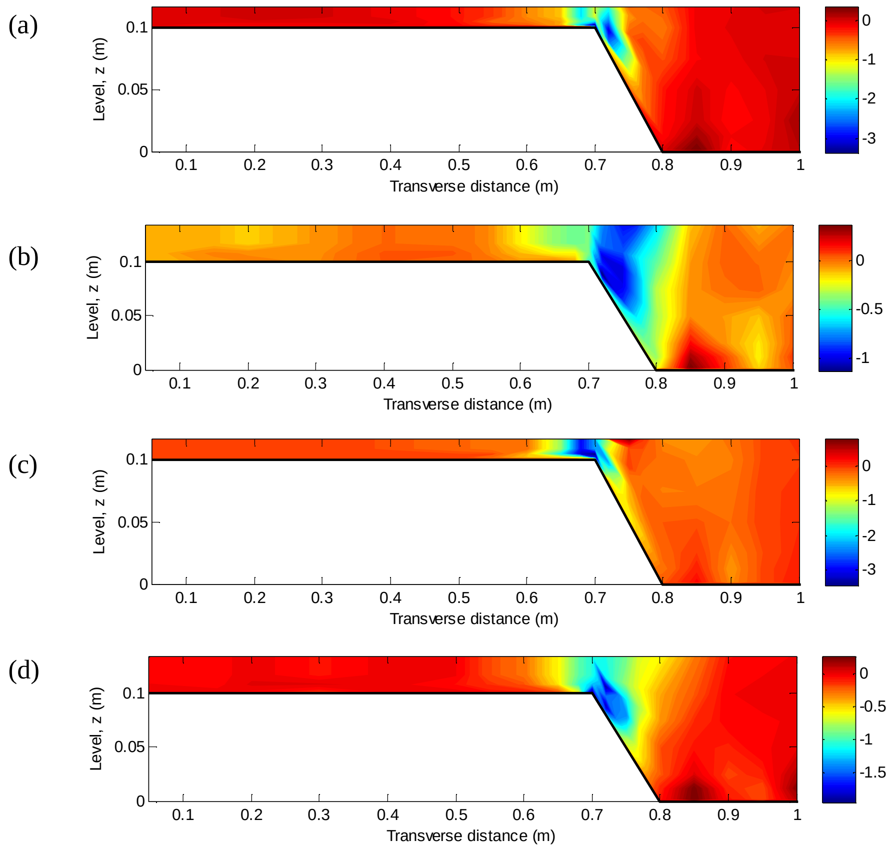

Figure 5. Reynolds stresses cross-section distribution (τ_{xy} in Pa). (a) hr015s; (b) hr03s; (c) hr015r and (d) hr03r

The depth averaged Reynolds stresses are presented in the Fig. 6.

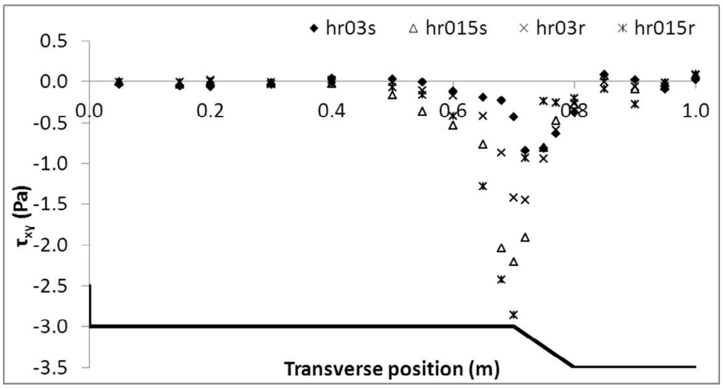

Figure 6. Depth averaged Reynolds stresses.

For all experiments the depth averaged Reynolds stresses is approximately zero for values of $y < 0{,}5$ and for $y > 0{,}85$. For both floodplain roughness, the values of the depth averaged Reynolds stresses near the interface increase from relative depth 0.15 to 0.3. The same occurs with the spreading of the shear layer.

5 Accuracy of the 1D methods

As referred above, several 1D methods have been developed to deal with the complexity of the compound channel flows. In the present work, the accuracy of these methods is assessed by comparing their predictions with the experimental results (*cf*. Table 1) obtained and presented in table 1.

The assessment of the accuracy by each method is based on the calculation errors computed by Eq. (15).

$$\text{Error}_{mc}\,(\%) = 100 \times \left(Q_{mc}^{Measured} - Q_{mc}^{Calculated}\right) / Q_{mc}^{Measured} \tag{15}$$

In which $Q^{Measured}$ stands for measured discharge and $Q^{calculated}$ for the discharge calculated by the method.

The complete results of these errors are presented in Table 2.

Table 2. Errors obtained by applying the different 1D method.

Flow reference	DCM	CH	DM	EDM	IDCM	WDCM
hr015S	-7%	0%	-2%	9%	-1%	4%
hr03S	-5%	3%	0%	4%	1%	6%
hr015R	-21%	-9%	-8%	12%	-11%	-2%
hr03R	-32%	-12%	-15%	1%	-15%	9%

These results are presented graphically in Fig. 7.

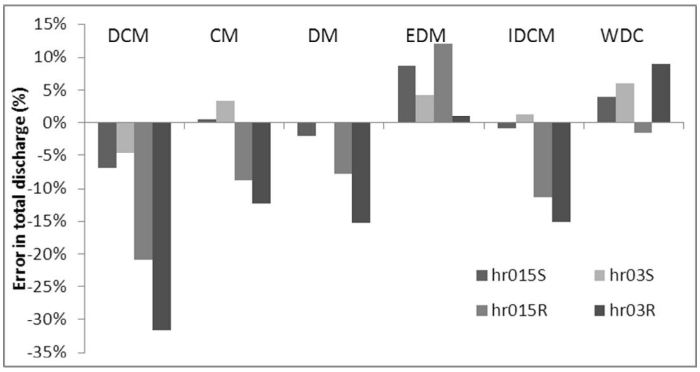

Figure 7. Error in the calculation of the discharges.

The DCM, assuming a simple division between the sub-sections of the entire channel, without considering the interaction between the subsections, leads to an over estimation of the discharge up to 7% when the channel has the same roughness along the entire perimeter. This over estimation is even bigger with rough floodplains with errors of up to 32%. The velocity reduction that the flow of floodplains causes in the main channel flow is not accounted in the main channel discharge. The errors decrease with the relative depth for smooth floodplain and increase with rough floodplains.

All methods allowed the improvement of the DCM results for the total discharge calculation. The results of the smooth floodplains conditions reveal good agreement with experiments. When the bottom has different roughness important discrepancies still occur.

6 CONCLUSIONS

The present work had two separate parts: an experimental study on the compound channel flow structure and the assessment of several one dimensional methods available in the literature.

The influence of the floodplain roughness on the streamwise velocities and on the Reynolds stresses is clear and can be observed in the Fig. 4 and 5. The artificial grass increases the floodplain and the main channel resistance to flow. The gradient of velocities increases when the floodplain has the artificial grass which leads to an increase of the Reynolds stresses (*cf.* Fig. 6).

Regarding the test of 1D methods, for smooth floodplains, good results on flow conveyance have been obtained by methods that take into account the momentum transfer. With the rough floodplains, the errors are relevant.

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